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LEADING CONTENTS

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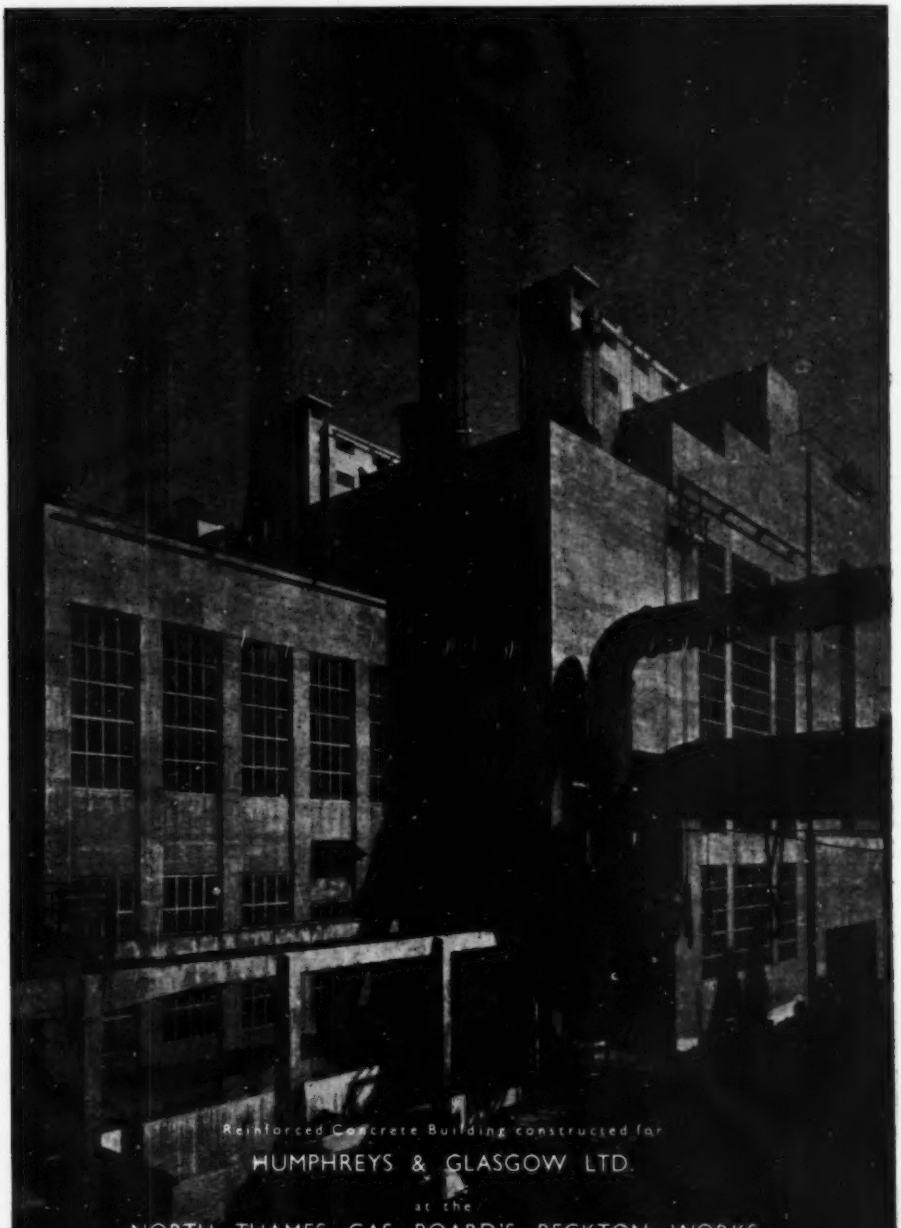
Reinforced Concrete Building constructed for
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Ferro-Concrete Bars on the Cooling Bed of the new Merchant Bar and Wire Rod Mill at Dalzell Steel Works.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume XLVI. No. 8. LONDON, AUGUST, 1951

EDITORIAL NOTES

Scarcities.

IT is estimated that in Great Britain this year the allocation of steel for reinforcement bars and wires for constructional work is about two-thirds of the requirements, and the precast concrete industry claims that orders are being accepted for only three-quarters of the quantity it needs. This is a striking commentary on the difference between the production of steel and the requirements, for at a time when licences are difficult to obtain for new buildings or even the repair of old ones it would not be justifiable to expect the Ministries and other authorities concerned to grant licences for steel-framed buildings if reinforced concrete structures needing much less steel would be equally satisfactory, or indeed nearly so. This is the state of affairs in the middle of the year 1951. It would be fruitless to try to guess what the position may be in the next few years if the plans for rearmament are put fully into practice and a still greater proportion of the production of steel is used for this purpose.

The construction industry has been accustomed to a scarcity of steel and most other building materials for more than twelve years. Indeed, there is a danger that scarcities have for so long been so large a part of our lives that they may be accepted as normal, and that no real effort will be made to find the true reason and to provide remedies. Reasons are given which appear to be sufficient, and which are generally accepted because to suggest the true reason would be unpopular. It is not, for example, sufficient to say that one of the causes of the shortage of steel is the lack of shipping to bring ore and scrap. At a time when ships are employed bringing coal from America, it is truer to say that the reason is the insufficient production of coal in Great Britain. A few weeks ago it was stated at the annual conference of the biggest trade union in the engineering industry that the rate of production per man had increased recently, and for this reason it was agreed to press for a reduction in the working hours to forty a week. That is, greater output should not be permitted, and as improved methods and machines come into operation the working hours should be reduced so that the previous rate of output is not exceeded. What an encouragement to use brains and spend money on new equipment! A noticeable trend of recent years is the extent to which forms of syndicalism are entering into various industries. In private as well as in nationalised industries there is an increasing tendency for employers and employed to make arrangements satisfactory to themselves but with little or no regard for the

rest of the people who have to pay higher prices or suffer the inconveniences that result from such arrangements.

A few weeks ago the Lord Privy Seal said that the Government depended upon increased output, and pleaded for a 20 per cent. greater effort from workers of all classes. This is the simple remedy for our present scarcities, namely that we all do more work each year. Everyone knows that in nearly every vocation people could do more work each year if they wished to do so or were allowed to do so, but few are willing to give expression to this cure for our scarcities because it would be unpopular to do so. Those who work in offices for as few as thirty-five hours a week and have up to a month's holiday a year would be loath to have this changed. Manual workers—or at least their leaders—look upon the five-days' week and a fortnight's holiday each year as a great step forward in what they call the "emancipation of the working classes", and an idle Saturday has become something sacrosanct which must not be interfered with however great the scarcities, hardships, inconvenience, and increased costs to them and the rest of the people—or, indeed, however much many of those who work five days a week would prefer to work also on Saturday mornings. Many of our scarcities are due to the fallacious claim that as much work could be done in five days as in five-and-a-half, ignoring the fact that in some of the most important industries the speed of the machine and the period it is working alone control the volume of production.

More leisure for all is very desirable if we can afford it, but it seems certain that in the past few years man has come to expect too many commodities and comforts for the amount of work he is prepared to do, with the result that the goods he produces are not sufficient for his needs, and the consequent higher prices and higher wages which buy ever fewer goods, because goods are produced by work and not by money. During the war and for a year or two afterwards we were told that the problem would be solved by scientists and technologists who would invent machines and processes that would vastly increase production without increased human effort. This has not happened. We were then told that management was at fault, but the visits to the United States of deputations from all sides of British industry have found that the lower production in Great Britain can be more often attributed to the desire to prevent unemployment even to the extent of refusing to use, or not fully using, labour-saving machinery, a disinclination to work more than one shift a day, and wage rates which give skilled men little more than labourers. The desire for more leisure is understandable, but insistence on being idle when we could be making the things we ourselves want is not. It is generally agreed that there would have been much unemployment in this country in the past few years if it were not for such fortuitous happenings as the gifts we have had from abroad, the keeping of nearly a million men in the armed forces, the swollen Civil Service, and now the work of rearmament. Such aids to employment will not for ever replace one another. We are very ready to listen to writers and talkers who claim that our economic problems can be solved by paper transactions such as altering the value of money, trading agreements, government control of our personal affairs and of industry, and other devices which do not suggest that people should do more work. Should we not realise that more work done now is the only remedy for our scarcities and the best insurance against unemployment and distress?

Wharf and Tunnels at Electricity Works.

THE new electricity power station for the British Electricity Authority at Bankside, London, will have oil-fired boilers. The oil will be discharged from tankers at a new wharf in the river Thames and will be pumped through pipes in a tunnel under the river bank and power-house to storage tanks at the rear of the power-house. Cooling water taken from the river at the wharf will flow through the intake tunnel to a suction chamber, and will be returned through the outfall tunnel which passes under the river and will discharge the water near the opposite bank. There is also an access tunnel from the wharf to the power-house.

The wharf (*Figs.* 1 to 5 and 8) is 290 ft. long and 38 ft. wide, and is designed to accommodate tankers carrying about 500 tons as well as smaller boats. The structure is in two parts; the upstream part is of cellular and mass construction and contains the water-intake works and the entrance to the access tunnel as in Section 2—2, and the downstream part is of open construction as in Section 1—1 (*Fig.* 3).

The deck, which is 8 ft. 6 in. above high-water and is generally a 12-in. reinforced concrete slab, carries buildings to accommodate the pumps and electrical equipment. The consulting civil engineers to the British Electricity Authority are Messrs. Mott, Hay and Anderson, and the contractors are Sir Robert McAlpine & Sons, Ltd.

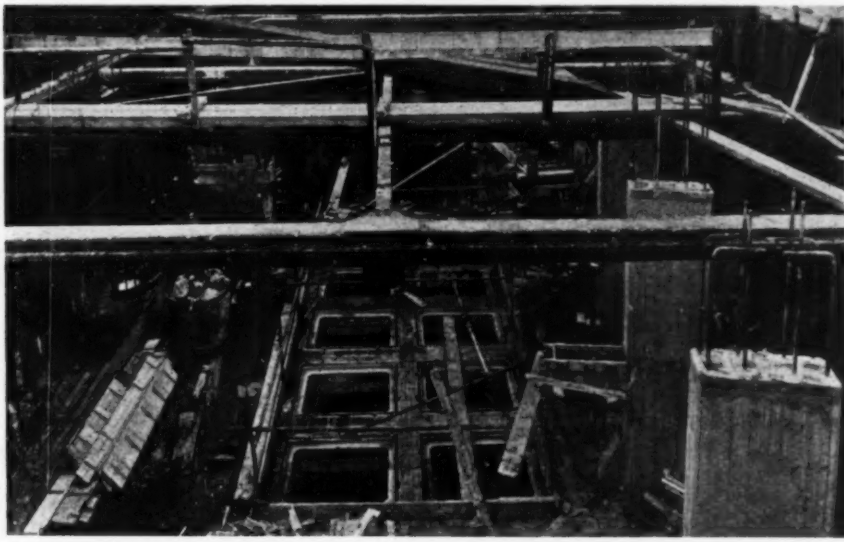


Fig. 1.—Excavation for Upstream Part of Wharf, showing Steel Struts in Cofferdam.

Intake Works.

The general level (– 24 ft. O.D.) of the foundation of the upstream part is 35 ft. 6 in. below high water and is generally a 7-ft. concrete slab laid on the clay. The shafts to the entrances of the access and intake tunnels extend about 28 ft. below the general foundation level. The reinforced concrete columns between the intake screens, the intake shaft, and well for the lift and stairs to the access tunnel are shown in *Figs.* 1 and 8.

This part of the wharf is being constructed in a steel sheet-pile cofferdam 96 ft. long and about 37 ft. wide and, when excavated, the depth was about 43 ft. The walings and struts were of welded

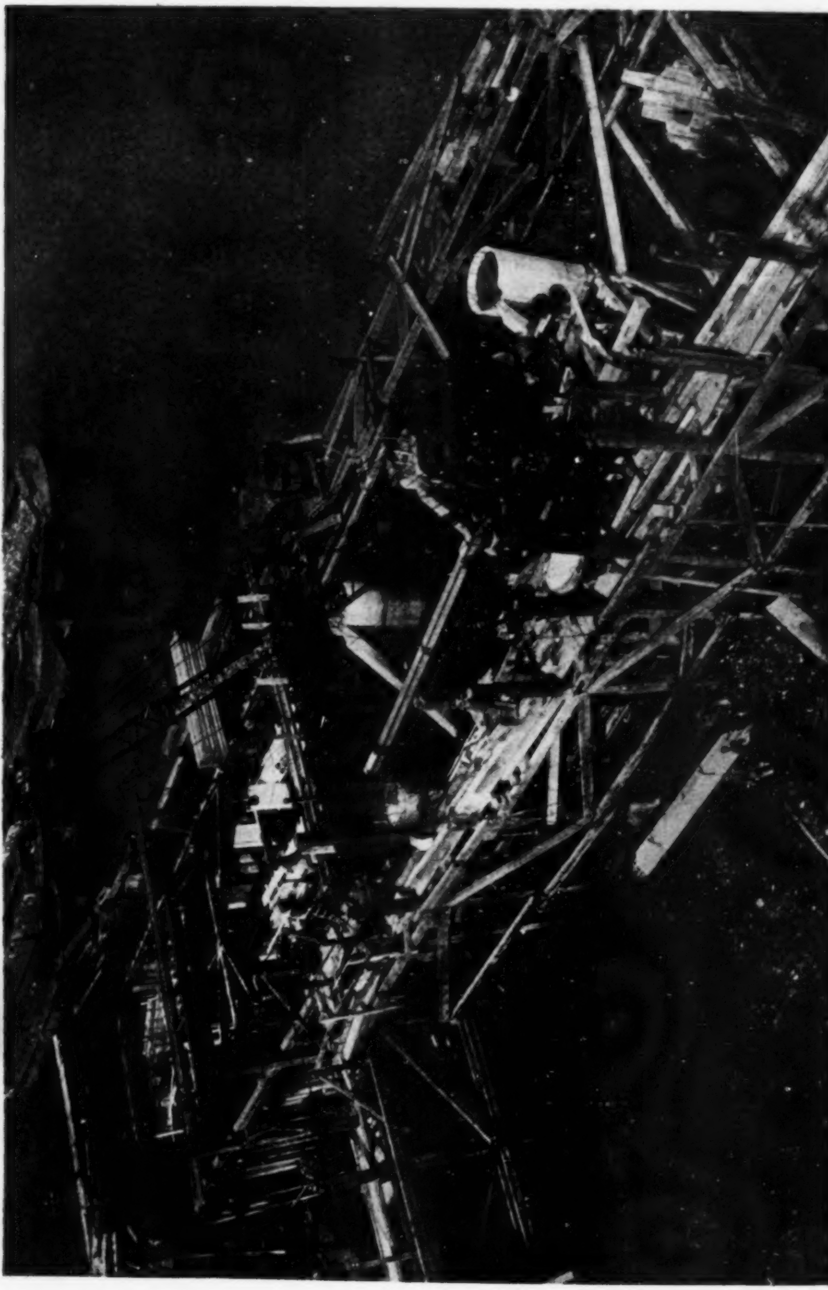


Fig. 2.—Wharf during Construction.

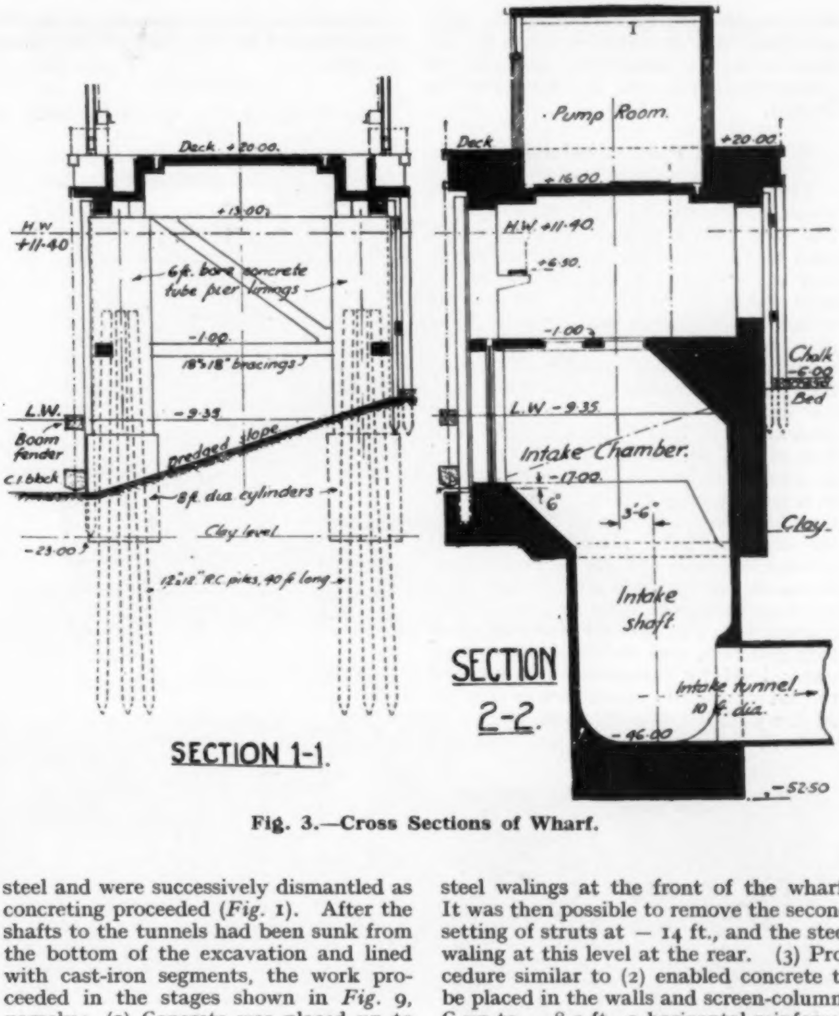


Fig. 3.—Cross Sections of Wharf.

steel and were successively dismantled as concreting proceeded (*Fig.* 1). After the shafts to the tunnels had been sunk from the bottom of the excavation and lined with cast-iron segments, the work proceeded in the stages shown in *Fig.* 9, namely: (1) Concrete was placed up to − 21·5 ft. and, when it had hardened, the lowest setting of walings and struts was removed. (2) The remainder of the concrete in the foundation slab up to − 17 ft. and in the walls for a height of 2 ft. up to − 15 ft. was placed. The part of the wall at the rear of the wharf is reinforced as a horizontal beam R_1 and acted temporarily as a waling; when the concrete had hardened, shortened struts S_1 were placed between the beam and vertical timbers V bearing against the steel walings at the front of the wharf. It was then possible to remove the second setting of struts at − 14 ft., and the steel waling at this level at the rear. (3) Procedure similar to (2) enabled concrete to be placed in the walls and screen-columns C up to − 8·5 ft., a horizontal reinforced concrete beam R_2 and a shortened strut S_2 being provided; the steel struts and rear waling at − 7·5 ft. could then be removed. (4) Concrete was placed up to + 1·0 ft., including a reinforced concrete slab at − 1·0 ft. and a beam R_3. Except for the front walings, the steel walings and struts at + 2·0 ft. were removed. (5) The concrete in the walls and piers up to + 13·0 ft. was placed. The walings and struts at + 14·0 ft. were then replaced by timber strutting and the

shuttering for the deck was erected. The cofferdam will be removed when the intake works are sufficiently advanced to permit this shaft and the tunnel to be flooded.

Open Part of Wharf.

The deck of the downstream part of the wharf is supported on eight pairs of piers. For each pier, a cylinder of steel sheet-piles of 8 ft. diameter was driven to the clay at about − 23·0 ft., the tops of the piles being at − 3·0 ft. The ground within the cylinders was excavated by grab, and four 12-in. square precast reinforced concrete piles were driven through the cylinders into the ground below each pier down to at least − 35·0 ft. The piles are 45 ft. long and are at an inclination of 1 in 40. They were driven by a $2\frac{1}{2}$-tons steam-hammer dropping 4 ft. 6 in. to a set of $\frac{1}{4}$ in. per blow. The cylinders were filled with concrete up to − 11·0 ft., the concrete being placed under water through a tremie. From − 11·0 ft. upwards the piers are constructed with spun reinforced concrete pipes (*Fig.* 4) in 6-ft. rings of 6 ft. 8 in. outside diameter, filled with concrete, and stabilised above low-water level by horizontal and inclined precast reinforced concrete braces (*Fig.* 3). At the junction of a pier and brace, the wall of the top pipe was broken into, and reinforcement bars projecting from the brace inserted into the cavity before filling the pier.

Fendering.

The fendering (*Fig.* 3) on the back of

Fig. 4.—Base of Pier.

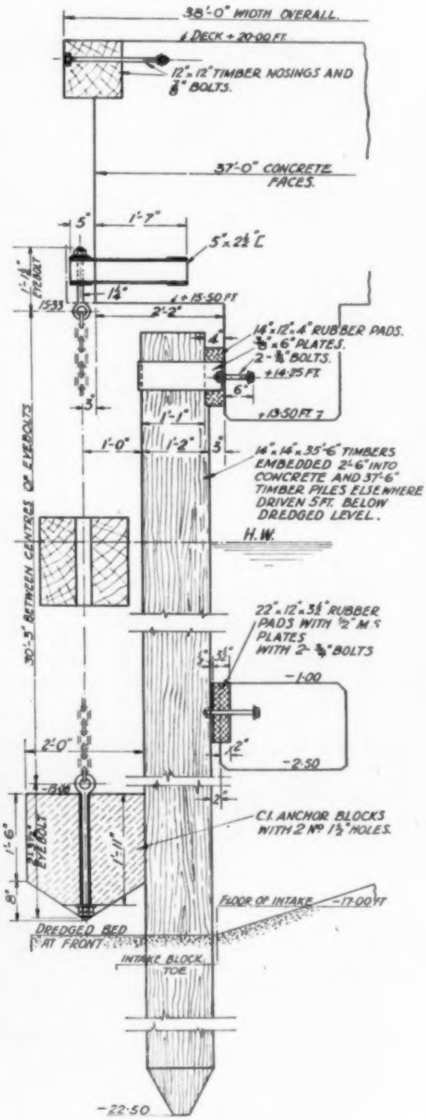


Fig. 5.—Fendering.

the wharf will comprise 12-in. square main timber piles at 6-ft. centres connected to the jetty only at the deck. Intermediate support will be provided by two 12-in. by 9-in. timber walings at + 1·5 ft. and + 11·0 ft. The walings will bear on an inner row of 12-in. square timber piles driven at 12-ft. centres midway between each pile and the concrete. The pile will bear against, but will not be secured to, the concrete brace at − 1·0 ft., to which will be attached a 3½-in. rubber pad protected by a ½-in. steel plate. The piles will be protected from direct blows from vessels by a horizontal 18-in. square floating timber boom, which will slide up and

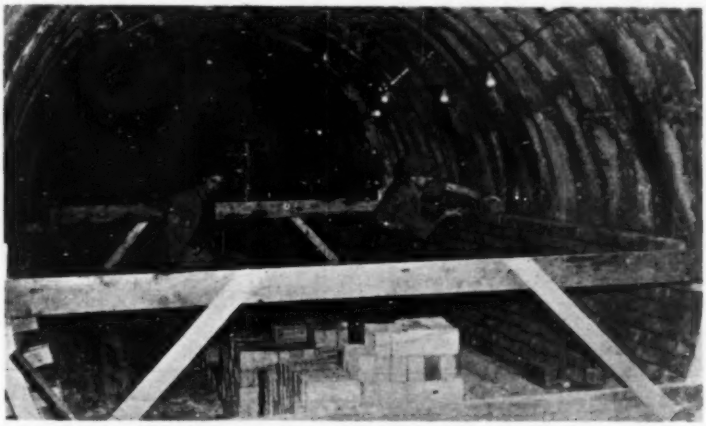


Fig. 6.—Brick Facing to Concrete Lining.

Fig. 7.—Precasting Lining for Tunnel.

pairs of main fender piles, and secured to the deck of the wharf and to the longitudinal reinforced concrete brace at − 1·0 ft.

The fendering on the front of the wharf, where the largest vessels will berth, will comprise 14-in. square vertical timber piles secured directly to the wharf only at the deck (*Figs.* 3 and 5), where a rubber pad 4 in. thick will be placed between down chains as the water rises and falls. The boom will be in lengths of about 24 ft., and two chains will be provided for each length. The chains will be suspended from the deck and be anchored in pairs to a block of cast iron 2 ft. wide, 2 ft. 2 in. deep, and generally 10 ft. long, and about 8 tons in weight.

At the rounded ends of the jetty, the fendering will comprise 12-in. by 8-in.

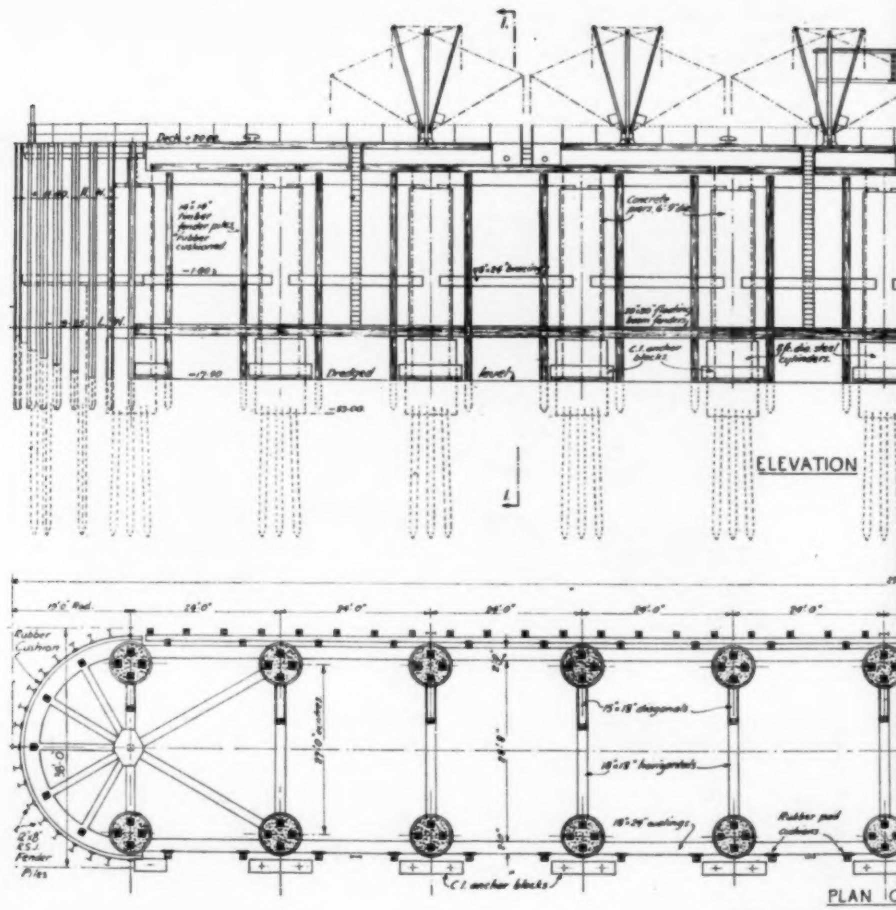


Fig. 8.—Elevati

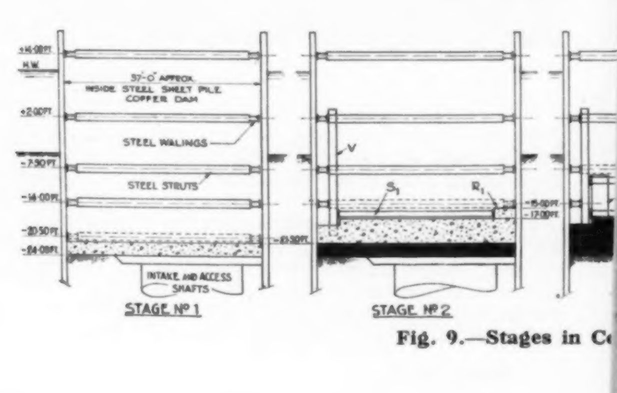
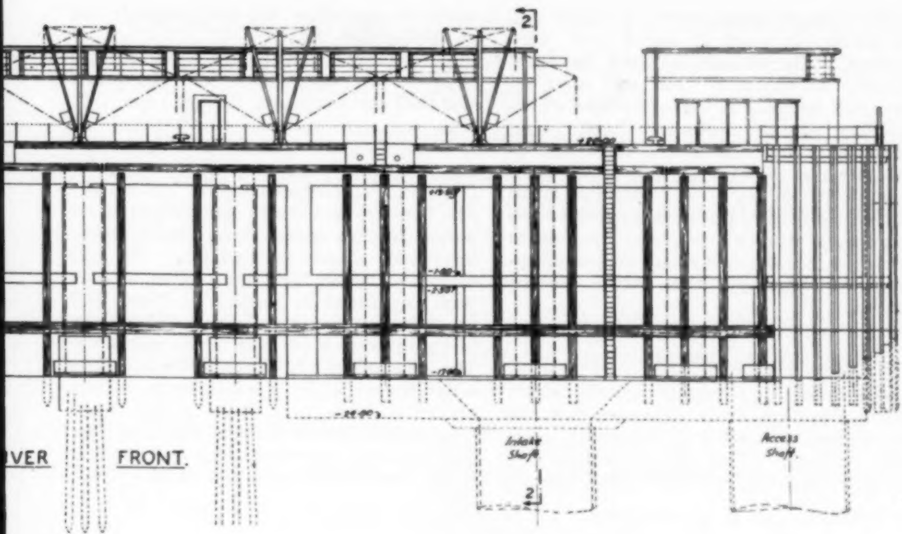
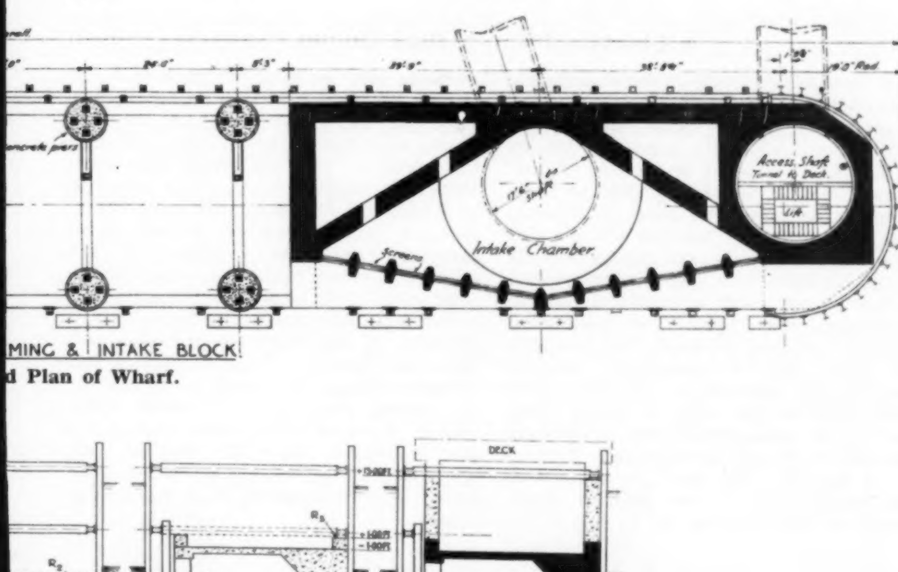
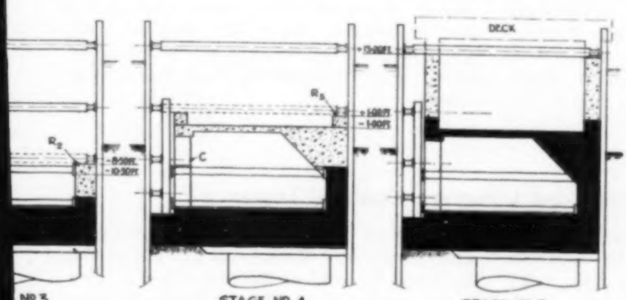


Fig. 9.—Stages in C





d Plan of Wharf.



tion of Intake Works.

vertical steel beams, most of which will be driven as piles at about 3-ft. 3-in. centres. The beams will not be fixed directly to the jetty, but will have on the back flange 2-ft. square plates which will bear against continuous hard-rubber bands at the levels of the deck and of the brace at – 1·0 ft. The upper band will be 12 in. by 9 in., and the lower 12 in. by 6 in. Forward movement of the beams will be prevented by a plate attached to the concrete and overlapping the edges of the plates on the beams.

The Tunnels.

The intake and outfall tunnels, which are in clay, are 10 ft. diameter and are constructed of cast iron segments lined with concrete. The intake tunnel was driven in free air, but the outfall tunnel, which passes under the river, was driven in compressed air.

The access tunnel serves as a footway between the power-house and the wharf and will accommodate the oil pipes. It is in clay and was driven in free air. It is 10 ft. diameter and lined with precast concrete segments and engineering bricks (*Fig.* 6). The segments were cast on the site (*Fig.* 7) on curved centering, the rebates for the interlocking joints being formed by shaped steel plates. The segments, of which there are twelve in each ring, are placed in position by hand, and grout is forced under pressure behind the lining through holes cast in the segments. The segments are of different lengths, and weigh from 200 lb. to 300 lb. each. They are 12 in. wide, and 5½ in. thick. Each ring of segments is reinforced by one ⅞-in. diameter bar, bent to a circle and grouted in the groove between the rings.

A Concrete Plant of High Output.

THE rate of concreting at the Bull Shoals dam, Arkansas, the construction of which is described in a recent number of "Concrete," is nearly 7,000 cu. yd. in 24 hours. The plant is capable of producing 8000 cu. yd. in 24 hours and comprises principally four 4-cu.-yd. mixers. The aggregates are brought seven miles by a belt-conveyor to crushing and screening plants at the site, and are transported to bins by conveyors in tunnels under the stockpiles. Two cylindrical steel bins, each of 138 tons capacity, are provided for each size of aggregate, thereby enabling the material in one bin to cool while that in the other is being used. Cooling is accomplished by continuous circulation through the aggregate of water at 35 deg. F. The sand is not cooled. The temperature of the concrete when placed is from 53 deg. to 63 deg. F. in summer and 42 deg. to 52 deg. F. in winter.

Above the mixers are four bins for coarse aggregates, one for sand and two for cement. The materials are automatically weigh-batched. The concrete is discharged into a hopper from which 4-cu.-yd. buckets are filled. A rail-wagon carrying four buckets, with space for another, is drawn to the site. A hammerhead crane (one of several travelling on a gantry parallel to the dam) places empty buckets in the unoccupied spaces and lifts a full bucket. When the fourth full bucket is taken, the wagon is returned to the mixing plant and this bucket, when empty, is placed on the next wagon. In this way more than 400 wagons each conveying 16 cu. yd. are unloaded daily when concrete is placed at the maximum rate.

A Large Reinforced Concrete Cooling Tower.

A REINFORCED concrete hyperbolic cooling tower recently completed at Stanlow refinery, Ellesmere Port, is thought to be the largest in the world. The tower, which is 341 ft. 6 in. high, will have a cooling capacity of 5,000,000 gallons per hour. The diameter at the base is 222 ft., at the neck 168 ft., and at the top 177 ft. The tower weighs about 20,000 tons and contains about 500 tons of reinforcement.

Bending Moments due to Temperature Changes in Continuous Reinforced Concrete Structures.

By ALBIN CHRONOWICZ.

THE distortion of a structure due to temperature changes may be due to different temperatures of two opposite faces of a member causing warping, or to the expansion or contraction of a member as a whole.

Different Temperatures of Opposite Faces of a Member.

In this case, referring to *Fig.* 1, if $T_2 - T_1 = \Delta T$, the extension of the hot face and contraction of the cold face is $\frac{\Delta T}{2}\varepsilon . ds$, which must equal $\frac{d_0}{2} d\theta$ if ε is the coefficient of thermal expansion. Hence $\frac{d\theta}{ds} = \frac{\Delta T . \varepsilon}{d_0}$. Since $\frac{ds}{R} = d\theta$, $\frac{d\theta}{ds} = \frac{1}{R} = \frac{M}{EI}$; therefore the bending moment M produced by the difference of temperature is $\frac{\Delta T . EI\varepsilon}{d_0}$. Restraining bending moments of the magnitude of M are required to produce a curvature to counteract the curvature due to the difference of temperature, and have the effect of end-fixity producing tension at the cold face.

It can generally be assumed that a reinforced concrete member cracks if it is subjected to a temperature exceeding 60 deg. F. Therefore the evaluation of the restraining bending moments should be based on the cracked section, neglecting in the calculation of the moment of inertia I that part of the concrete which is in tension. Therefore, referring to *Fig.* 2a, $I = \frac{b(n_1 d)^3}{3} + mA_s(d - n_1 d)^2$.

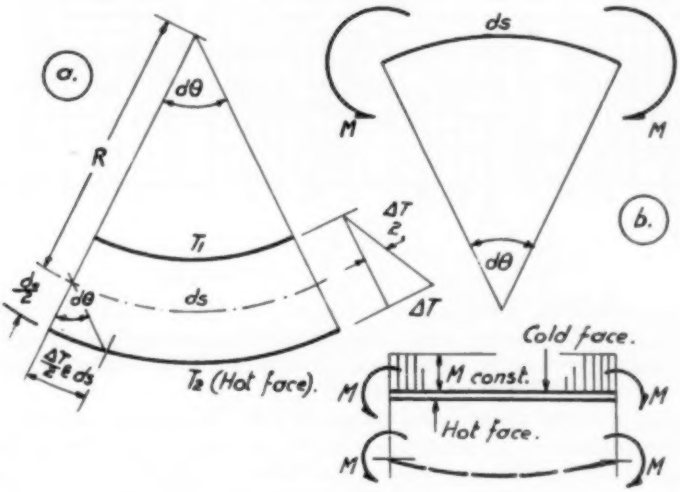


Fig. 1.

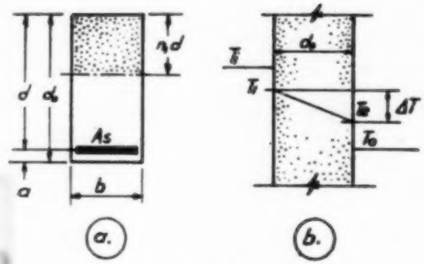


Fig. 2.

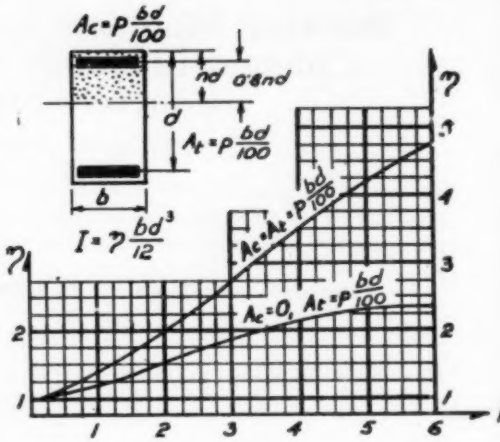


Fig. 3.

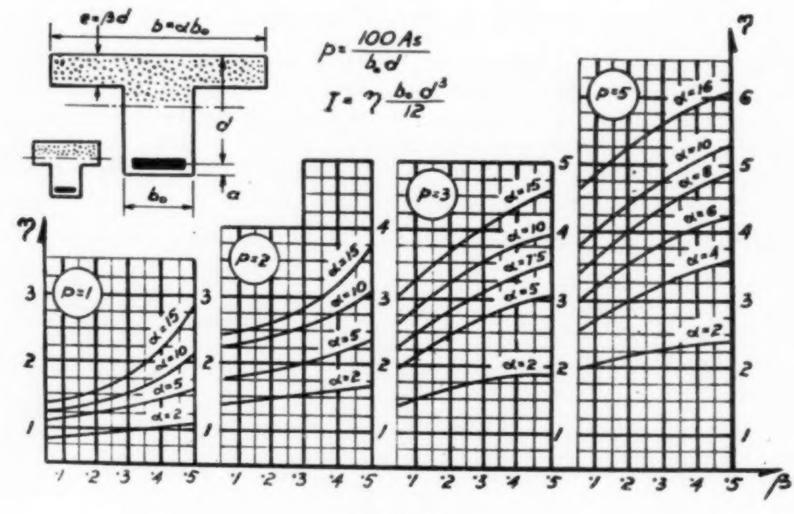


Fig. 4.

The curves in *Figs.* 3 and 4 give values of I in terms of bd^2 for "cracked" sections of rectangular beams (with and without compressive reinforcement) and tee-beams, assuming $m = 15$. For a wall or slab of a total thickness d_0, and b of 12 in., the average value of I is about d_0^3.

Assuming ε to be about 0·000006 per degree Fahrenheit and the elastic modulus E of concrete to be about 2,000,000 lb. per square inch, the product εE is about 12. Assuming also a uniform flow of heat through the wall, if T_i and T_o are the air temperatures (*Fig.* 2*b*), ϕ the coefficient of heat transmission of the

wall, and λ the coefficient of conductivity of the material of the wall, d_0 the thickness (feet) of the wall, and α_i and α_o the coefficients of radiation at the inside and outside faces respectively, the following apply to 1 sq. ft. of the face of the wall:

Total amount of heat lost through wall: $Q_1 = \phi(T_i - T_o)$.
Amount of heat entering the hotter face: $Q_2 = \alpha_i(T_i - T_1)$

" " " passing through the wall: $Q_3 = \dfrac{\lambda}{d_o}(T_1 - T_2)$

" " " leaving the cooler face: $Q_4 = \alpha_o(T_2 - T_o)$

For a constant flow of heat, $Q_1 = Q_2 = Q_3 = Q_4$. Since $T_1 - T_2 = \Delta T$, combining the foregoing expressions gives $\Delta T = \dfrac{T_i - T_o}{\frac{1}{\phi}} \cdot \dfrac{d_0}{\lambda}$.

If the wall comprises layers of different materials of thickness $d_1, d_2, d_3, \ldots d_n$ (ft.), $\dfrac{1}{\phi} = \dfrac{1}{\alpha_i} + \dfrac{d_1}{\lambda_1} + \dfrac{d_2}{\lambda_2} + \ldots \dfrac{d_n}{\lambda_n} + \dfrac{1}{\alpha_o}$.

Common values of λ, when the units are feet and degrees Fahrenheit, are: For concrete, 1; gypsum, 0·3; brick, 0·5; slag-wool, 0·1; cork, 0·06. The coefficient of radiation ($\alpha_i = \alpha_o$) for concrete is about 2.

EXAMPLE NO. 1.—Determine the restraint bending moments due to the difference of temperature on two sides of a reinforced concrete slab ($\lambda = 1$; $\alpha_i = \alpha_o = 2$; $\varepsilon E = 12$), for which $d_0 = 0{\cdot}5$ ft., $T_i = 120$ deg., and $T_o = 30$ deg. F. By substitution, $\dfrac{1}{\phi} = \dfrac{1}{2} + \dfrac{0{\cdot}5}{1} + \dfrac{1}{2} = 1{\cdot}5$, and $\Delta T = \dfrac{120 - 30}{1{\cdot}5} = 30$ deg. F. Since $I = d_0^3$ (approximately) $= 6^3 = 216$ in.^{4}, the fixed-end bending moment due to the difference of temperature is given by $M = \dfrac{30 \times 12 \times 216}{6} = 13{,}000$ in.-lb. Adjustments can be made by the usual moment-distribution method.

Members Subjected to Uniform Change of Temperature.

Assume that the members of the frame in *Fig.* 5 are subjected to expansion due to a rise of temperature. By the usual process of strain-energy analysis,

$$M_{AB}^x = Hx; \quad \frac{\delta M}{\delta H} = x; \quad \frac{\delta U}{\delta H} = \int_0^h \frac{M\frac{\delta M}{\delta H}\,dx}{EI}.$$

$$M_{BC}^x = Hh; \quad \frac{\delta M}{\delta H} = h; \quad \frac{\delta U}{\delta H} = \int_0^l \frac{M\frac{\delta M}{\delta H}\,dx}{EI}.$$

Hence
$$\varepsilon Tl = \frac{\delta U}{\delta H} = \frac{2H}{EI}\int_0^h x^2\,dx + \frac{Hh^2}{EI}\int_0^l dx,$$

$$\frac{2Hh^3}{3} + Hh^2 l = \varepsilon TlEI,$$

from which
$$H = \frac{3\varepsilon TlEI}{h^2(3l + 2h)}, \text{ and } M_{BA} = \frac{3\varepsilon TlEI}{h(3l + 2h_3)}.$$

By moment distribution (*Fig.* 5), the fixed-end bending moments are $\frac{3EI}{h^2}\varepsilon\frac{l}{2}T$. Because of symmetry the distribution factors are:

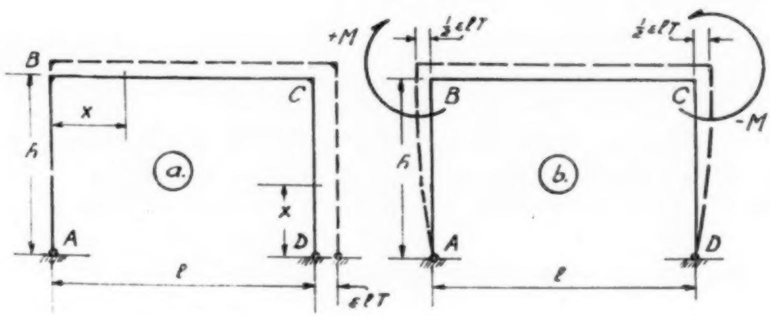
$$\gamma_{BA} = \frac{\frac{3}{4h}}{\frac{3}{4h} + \frac{1}{2l}} = \frac{3l}{3l + 2h}, \text{ and } \gamma_{BC} = 1 - \gamma_{BA} = \frac{2h}{3l + 2h}.$$

The moment-distribution operations are shown in *Fig.* 5.

It follows that thermal extension of the members of a frame can be regarded as displacement of the respective joints and the resulting fixed-end bending moments are distributed in the usual way.

EXAMPLE NO. 2.—The reinforced concrete duct (*Fig.* 6) for the passage of hot gases will be analysed. The frame constants are $\frac{I_{BC}}{I_{AB}} = \frac{0{\cdot}5^3}{0{\cdot}33^3} = 3{\cdot}4$. Since relative values of I only are needed, $I_{AB} = 1$ and $I_{BC} = 3{\cdot}4$. Therefore the stiffness factors are: $K_{AB} = \frac{1}{8} = 0{\cdot}125 =$, say, 1 unit; $K_{BC} = \frac{3{\cdot}4}{14} = 0{\cdot}243$ unit; $\frac{K_{BC}}{2} = 0{\cdot}12 = 1$ unit; and, because of the symmetry of the frame, $\gamma_{BC} = 0{\cdot}5$.

(i) Distortion due to differences of temperature.—For AB (*Fig.* 6*c*), $T_i - T_o = 240$ deg. F. Therefore $\frac{1}{\phi} = \frac{1}{2} + \frac{0{\cdot}33}{1} + \frac{1}{2} = 1{\cdot}33$, and ΔT_{AB} is



	BA	BC	
	$\frac{3l}{3l+2h}$	$\frac{2h}{3l+2h}$	Moment-distribution factors.
	$\frac{3EI}{h^2}\cdot\frac{\varepsilon l}{2}T$		Fixed-end moments.
	$-\frac{3EI}{h^2}\cdot\frac{\varepsilon l}{2}T\cdot\frac{3l}{3l+2h}$	$-\frac{3EI}{h^2}\cdot\frac{\varepsilon l}{2}T\cdot\frac{2h}{3l+2h}$	Distribution.
	$\frac{3\varepsilon T l EI}{h(3l+2h)}$	$-\frac{3\varepsilon T l EI}{h(3l+2h)}$	Final bending moments.

c.

Fig. 5.

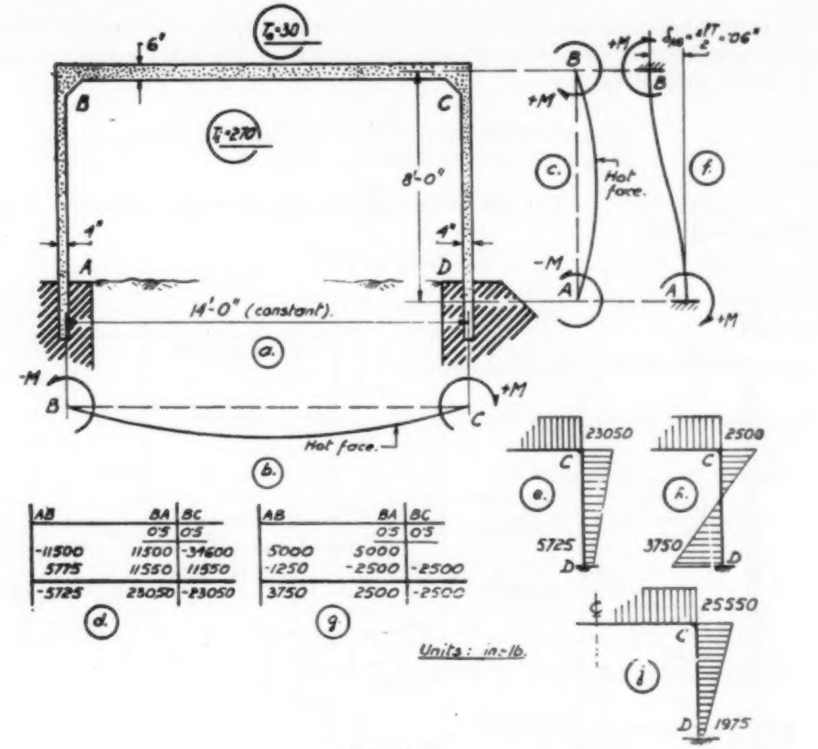


Fig. 6.

$\dfrac{240}{1\cdot 33} \times 0\cdot 33 = 60$ deg. F. Also $I = 4^3 = 64$ in.^{4} The fixed-end bending moment due to difference of temperature is therefore given by

$$M = \frac{60 \times 12 \times 64}{4} = 11{,}500 \text{ in.-lb. approximately.}$$

For BC (*Fig.* 6*b*), $\dfrac{1}{\phi} = \dfrac{1}{2} + \dfrac{0\cdot 5}{1} + \dfrac{1}{2} = 1\cdot 5$; $I = 6^3 = 216$ in.^{4}; and $\Delta T = \dfrac{240}{1\cdot 5} \times 0\cdot 5 = 80$ deg. F. Therefore $M = \dfrac{80 \times 12 \times 216}{6} = 34{,}600$ in.-lb. The moment-distribution operations and the resulting bending-moment diagram are shown in *Fig.* 6*d* and *e*.

(ii) Distortion due to expansion of the members.—The average rise of temperature T is $\frac{1}{2}(T_i - T_o) = 120$ deg. F. Therefore

$$2\delta = \epsilon l T = 0\cdot 000006 \times 14 \times 120 \times 12 = 0\cdot 121 \text{ in.,}$$

and the fixed-end bending moment (*Fig.* 6*f*) is given by

$$M_{AB} = \frac{6EI\delta}{h^2} = \frac{6 \times 2{,}000{,}000 \times 64 \times 0\cdot 06}{96^2} = 5000 \text{ in.-lb.}$$

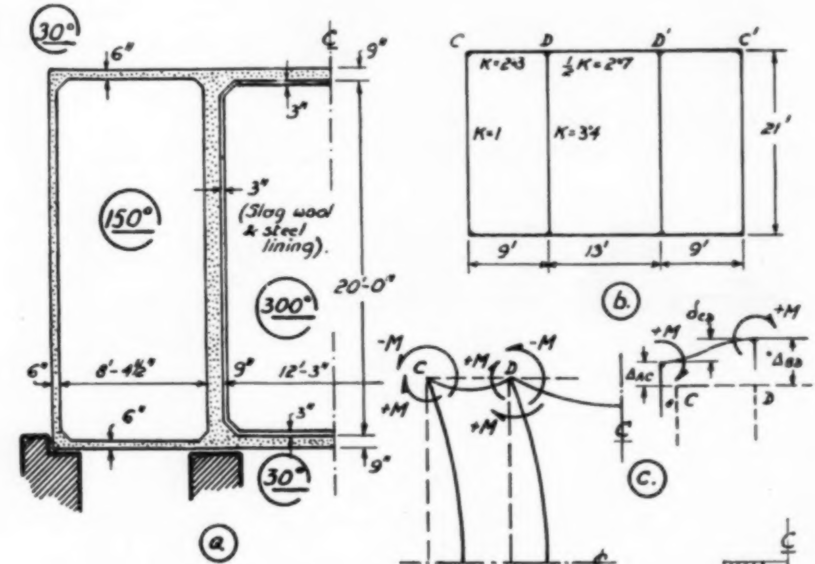
The moment-distribution operations are given in *Fig.* 6*g* and the bending-moment diagram in *Fig.* 6*h*. Combining the effects of (i) and (ii), the final bending moments, as shown in *Fig.* 6*j*, are

$$M_{BA} = +\ 23{,}050 + 2500 = +\ 25{,}550 \text{ in.-lb.},$$

and

$$M_{AB} = -\ 5725 + 3750 = -\ 1975 \text{ in.-lb.}$$

EXAMPLE No. 3.—The reinforced concrete testing-tunnel shown in *Fig.* 7 will be analysed.



BB'	BD	BA	AB	AC	CA	CD	DC	DB	DD'	
0·33	0·4	0·27	0·7	0·3	0·3	0·7	0·27	0·4	0·33	
450	-250	-173	173	-173	173	-173	173	250	-450	F.E.Ms.
						455	455			
-9	-11	-7	-3					-5		
			1	2	1					
				-67	-134	-312	-156			
	-51	23	47	20	10	-35	-70	-102	-85	
				3	7	18	9			
			-1	-2	-1	1				
9	11	8	4					5		
		-1	-3	-1						
	-2					-2	-4	-5	-5	
1	2					2	1	-1		
451	-301	-150	218	-218	56	-56	398	142	-540	

d.

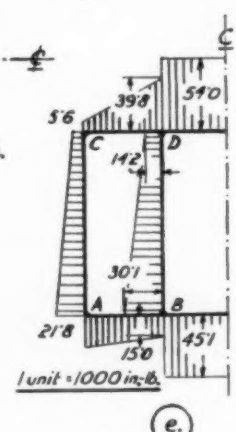


Fig. 7.

Frame constants :

$$I_{AC} = 6^3 = 216 \text{ in.}^4;\ K_{AC} = \frac{216}{21} = 10{\cdot}3 = 1.$$

$$I_{AB} = I_{CD} = 216 \text{ in.}^4;\ K_{CD} = \frac{216}{9} = 24 = 2{\cdot}3 = K_{AB}.$$

$$I_{DB} = 9^3 \simeq 730 \text{ in.}^4;\ K_{DB} = \frac{730}{21} = 34{\cdot}8 = 3{\cdot}4.$$

$$I_{BB'} = I_{DD'} \simeq 730 \text{ in.}^4;\ \frac{K_{DD'}}{2} = 0{\cdot}5 \times \frac{130}{13} = 28{\cdot}1 = 2{\cdot}7 = \frac{K_{BB'}}{2}.$$

Moment-distribution factors :

$$\gamma_{CA} = \frac{1}{3{\cdot}3} \simeq 0{\cdot}3;\ \gamma_{CD} = 0{\cdot}7;\ \gamma_{DC} = \frac{2{\cdot}3}{8{\cdot}4} = 0{\cdot}27;\ \gamma_{DB} = \frac{3{\cdot}4}{8{\cdot}4} = 0{\cdot}4;\ \gamma_{DD'} = 0{\cdot}33.$$

(i) Fixed-end bending moments due to distortion due to differences of temperature.

For members CA and CD, $T_i - T_o = 120$ deg. F., $I = 216$ in.^{4} $\frac{1}{\phi} = \frac{1}{2} + \frac{0{\cdot}5}{1} + \frac{1}{2} = 1{\cdot}5$, and $\Delta T = \frac{120}{1{\cdot}5} \times 0{\cdot}5 = 40$ deg. F. The fixed-end bending moment is $\frac{40 \times 12 \times 216}{6} = 17{,}300$ in.-lb. approximately.

For member BD, $T_i - T_o = 150$ deg. F.; $I = 730$ in.^{4}. The value of λ for steel is large, and $\frac{d}{\lambda}$ for steel sheets is assumed to be negligible; α for the face of steel is assumed to be $1{\cdot}5$. Hence, $\frac{1}{\phi} = \frac{1}{2} + \frac{0{\cdot}75}{1} + \frac{0{\cdot}25}{0{\cdot}1} + \frac{1}{1{\cdot}5} = 4{\cdot}4$ approximately, and $\Delta T = \frac{150}{4{\cdot}4} \times 0{\cdot}75 = 25{\cdot}5$ deg. F. The fixed-end bending moment is $\frac{25{\cdot}5 \times 12 \times 730}{9} = 25{,}000$ in.-lb. approximately.

For member DD′, $T_i - T_o = 270$ deg. F., $I = 730$ in.^{4}, $\frac{1}{\phi} = 4{\cdot}4$, $\Delta T = \frac{270}{4{\cdot}4} \times 0{\cdot}75 = 46$ deg. F., and the fixed-end bending moment is

$$\frac{46 \times 12 \times 730}{9} = 45{,}000 \text{ in.-lb. approximately.}$$

(ii) Fixed-end bending moment due to distortion due to expansion of the members.

The members of the frame have freedom of horizontal movement, but the vertical expansion of members BD and B′D′ is restricted because of the smaller rise of temperature of members AB and A′B′, a condition which produces bending of members CD and C′D′. From *Fig.* 7c, $\delta_{CD} = \Delta_{BD} - \Delta_{AC}$. For Δ_{AC}, $T = \frac{150 - 30}{2} = 60$ deg. F. For Δ_{BD}, $T = \frac{300 + 150}{2} - 30 = 195$ deg. F.

Hence, for δ_{CD}, $T = 195 - 60 = 135$ deg. F., and

$$\delta_{CD} = 135 \times 0{\cdot}000006 \times 252 = \text{about } 0{\cdot}2 \text{ in.}$$

The fixed-end bending moment at C in CD is

$$\frac{6 \times 216 \times 0{\cdot}2 \times 2{,}000{,}000}{108^2} = 44{,}500 \text{ in.-lb.}$$

The moment-distribution operations are given in *Fig. 7d* and the diagram of the final bending moments in *Fig. 7e*.

EXAMPLE No. 4.—The elevated smoke-precipitator structure in *Figs.* 8 and 9 will be analysed.

Frame constants (*Fig. 9a*). Assume that for members AB, BB′, and BC, $p = 1{\cdot}75$ per cent. and $a = 2$ in. Then, from the curves in *Fig.* 3, $I = 1{\cdot}8 \times \frac{b \times 14^2}{12}$. Therefore $I_{AB} = 7350$ in.^{4}, $I_{BB'} = 6150$ in.^{4}, and $I_{BC} = 5510$ in.^{4} Therefore $\frac{3}{4}K_{AB} = 0{\cdot}75 \times \frac{7350}{22{\cdot}75} = 243 =$, say, $1{\cdot}59$ units;

$\frac{1}{2}K_{BB'} = 0{\cdot}5 \times \frac{6150}{20} = 153 = 1$ unit; and $K_{BC} = \frac{5510}{16{\cdot}75} = 329 = 2{\cdot}15$ units.

In these members, the flange is on the tension side or coincides with the neutral

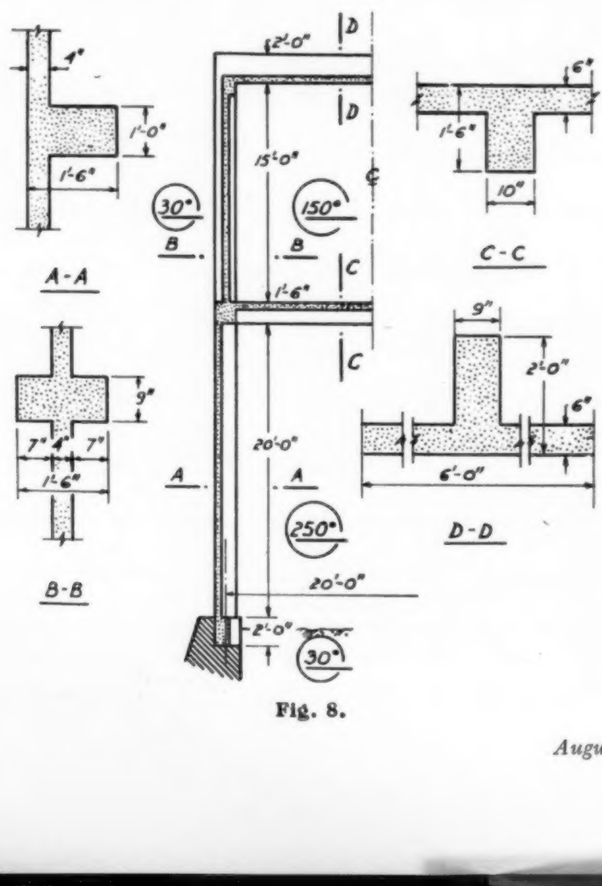
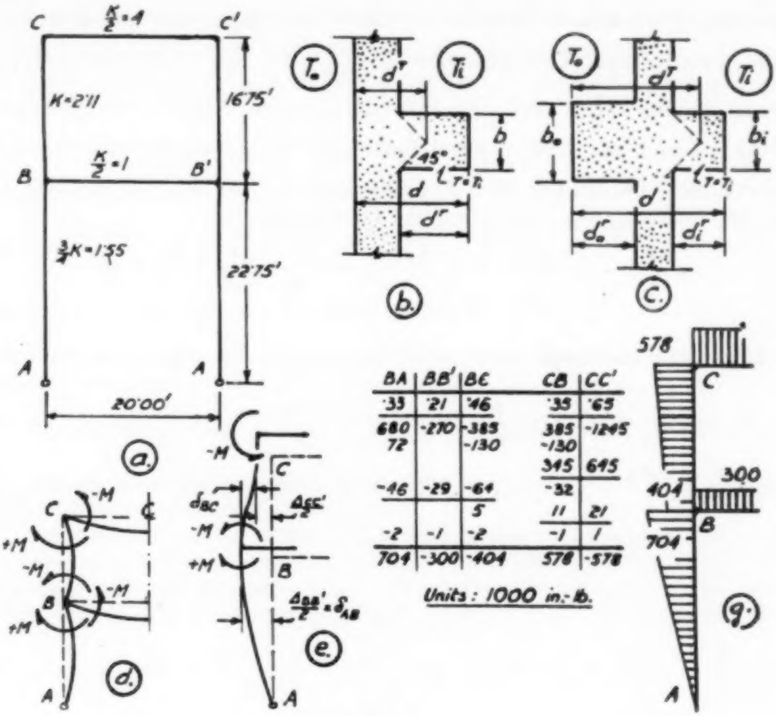


Fig. 8.



BA	BB'	BC	CB	CC'
·33	·21	·46	·35	·65
680	-270	-385	385	-1245
72		-130	-130	
			345	645
-46	-29	-64	-32	
		5	11	21
-2	-1	-2	-1	1
704	-300	-404	578	-578

Fig. 9.

axis; therefore the flange is neglected in the calculation of the moment of inertia and stiffness of the member. In member CC′, the flange is on the compression side and must be included in the evaluation of I and K. Assuming $p = 3$ per cent. and $a = 2$ in., for $\alpha = \dfrac{6}{0{\cdot}75} = 8$ and $\beta = \dfrac{6}{22} = 0{\cdot}27$, from the curves in *Fig.* 4, $I_{CC'} = 3{\cdot}1 \times \dfrac{9 \times 22^3}{12} = 24{,}800$ in.^{4}, and $\frac{1}{2}K_{CC'} = 0{\cdot}5 \times \dfrac{24{,}800}{20} = 620 = 4{\cdot}05$ units. The amount of reinforcement assumed is based on a preliminary design for static loads only.

Moment distribution factors:

$$\text{At joint B, } \gamma_{BA} = \frac{1{\cdot}59}{4{\cdot}74} = 0{\cdot}33,\ \gamma_{BB'} = \frac{1}{4{\cdot}74} = 0{\cdot}21, \text{ and } \gamma_{BC} = 1-(0{\cdot}33+0{\cdot}21) = 0{\cdot}46.$$

$$\text{At joint C, } \gamma_{CC'} = \frac{4{\cdot}05}{6{\cdot}20} = 0{\cdot}65, \text{ and } \gamma_{CB} = 1-0{\cdot}65 = 0{\cdot}35.$$

(i) Fixed-end bending moments due to differences of temperature.

It is necessary first to consider the heat transference through a tee-member, such as AB (*Fig.* 9*b*). The total loss of heat Q_1 across a member of superficial area A is $\phi A\ (T_i - T_o)$. The heat Q_2 entering the member is $\alpha_i n_i A(T_i - T_1)$ if n_i

is the ratio of the area of the surface of the rib corresponding to unit area ($b \times 1$) of the opposite face. Therefore $n_i = \frac{2d_r + b}{b}$. The heat Q_4 leaving the outside face is $\alpha_o A(T_2 - T_o)$, and the heat Q_3 passing through the member is $\frac{\lambda}{d_T} A(T_1 - T_2)$, where it is assumed that d_T is the "gradient depth" and the temperature of the cracked rib is T_i approximately. As before, $Q_1 = Q_2 = Q_3 = Q_4$ and, by combining the expressions for these quantities,

$$\Delta T = T_1 - T_2 = \frac{T_i - T_o}{\frac{1}{\phi}} \cdot \frac{d_T}{\lambda}, \text{ and } \frac{1}{\phi} = \frac{1}{n_i \alpha_i} + \frac{d_T}{\lambda} + \frac{1}{\alpha_o}.$$

In the present example, for member AB, $n_i = 1 + \frac{28}{12} = 3{\cdot}33$; $d_T = 0{\cdot}83$ ft.; $T_i - T_o = 220$ deg. F.; $\frac{1}{\phi} = \frac{1}{3{\cdot}33 \times 2} + \frac{0{\cdot}83}{1} + \frac{1}{2} =$ about $1{\cdot}48$; and $\Delta T = \frac{220 \times 0{\cdot}83}{1{\cdot}48} = 123$ deg. F. The fixed-end bending moment M_{BA} is $\frac{123 \times 12 \times 7350}{16} = 680{,}000$ in.-lb.

For member BB′, $T_i - T_o = 100$ deg. F.; $d_T = 0{\cdot}92$ ft.; $n_i = 1 + \frac{2{\cdot}4}{10} = 3{\cdot}4$; $\frac{1}{\phi} = \frac{1}{3{\cdot}4 \times 2} + \frac{0{\cdot}92}{1} + \frac{1}{2} = 1{\cdot}57$; and $\Delta T = \frac{100 \times 0{\cdot}92}{1{\cdot}57} = 58{\cdot}5$ deg. F. The fixed-end bending moment $M_{BB'}$ is $\frac{58{\cdot}5 \times 12 \times 6150}{16} = 270{,}000$ in.-lb.

For member BC (*Fig. 9c*), $T_i - T_o = 120$ deg. F. Because of the different shape of the cross section,

$$b = \frac{b_i + b_o}{2}; \quad n_i = \frac{2d_{ri} + b_i}{b}; \quad n_o = \frac{2d_{ro} + b_o}{b}; \quad \frac{1}{\phi} = \frac{1}{n_i \alpha_i} + \frac{d_T}{\lambda} + \frac{1}{n_o \alpha_o}.$$

Substituting the numerical values $b_i = b_0 = b = 9$ in., $d_{ri} = d_{r0} = 7$ in., $n_i = n_o = 1 + \frac{14}{9} = 2{\cdot}56$, and $d_T = 1{\cdot}3$ ft., in these expressions, $\frac{1}{\phi} = 1{\cdot}69$ and $\Delta T = \frac{120 \times 1{\cdot}3}{1{\cdot}69} = 93$ deg. F. The fixed-end bending moment M_{BC} is $\frac{93 \times 12 \times 5510}{16} = 385{,}000$ in.-lb.

For member CC′, $T_i - T_o = 120$ deg. F.; $d_T = d = 2$ ft.; $n_o = 1 + \frac{2 \times 1{\cdot}5}{0{\cdot}75} = 5$; $\frac{1}{\phi} = \frac{1}{2} + \frac{2}{1} + \frac{1}{2 \times 5} = 2{\cdot}6$; and $\Delta T = \frac{120 \times 2}{2{\cdot}6} = 92$ deg. F. The fixed-end bending moment $M_{CC'}$ is $\frac{92 \times 12 \times 24{,}800}{22} = 1{,}245{,}000$ in.-lb.

The directions of action of these bending moments are shown in *Fig.* 9*d*.

(ii) Fixed-end moments due to expansion of the members.

In *Fig.* 9*e*, $\delta_{AB} = \frac{1}{2}\Delta_{BB'}$, where δ_{AB} is the displacement of the joints of member AB and $\delta_{BB'}$ is the extension of member BB′ due to the rise of temperature T of $\frac{250 + 150}{2} - 30 = 170$ deg. F. Hence $\delta_{AB} = \frac{1}{2}\varepsilon T l_{BB'}$, and the fixed-end bending moment is given by

$$M_{BA} = \frac{3EI_{AB}\delta_{AB}}{l^2_{AB}} = \frac{3I_{AB}Tl_{BB'}\varepsilon E}{2l^2_{AB}} = \frac{3\times 7350 \times 170 \times 240 \times 12}{2 \times 273^2} = 72{,}000 \text{ in.-lb.}$$

Also, $\delta_{BC} = \frac{1}{2}(\Delta_{BB'} - \Delta_{CC'})$. For member CC′, T is $\frac{150 - 30}{2} = 60$ deg. F. Also $\Delta_{CC'} = \varepsilon \times 60 \times 240$, and $\Delta_{BB'} = \varepsilon \times 170 \times 240$. Hence $\delta_{BC} = 13{,}200\varepsilon$. The fixed-end bending moments are given by

$$M_{BC} = M_{CB} = \frac{6 \times 5510 \times 13{,}200 \times \varepsilon E}{201^2} = 130{,}000 \text{ in.-lb., if } \varepsilon E = 12.$$

The moment-distribution operations are shown in *Fig.* 9 and the diagram of the final bending moments in *Fig.* 9*g*.

These calculations are based on assumed amounts of reinforcement. When the final sections have been designed, the actual values of p and I should be compared with the assumed values, and if there are large discrepancies the calculations should be repeated.

Designs of Dams and Tunnels.

In a paper by Mr. C. M. Roberts, M.Inst.C.E., on "Fundamental Economics in Hydro-electric Design," published in the Journal of the Institution of Civil Engineers for April, 1951, comparative costs of dams and tunnels are given.

The first example is a dam with a maximum height of 134 ft. and a length along the crest 950 ft.; sound rock occurs near the surface, and a mound of rock at about the middle of the length of the dam makes a double-arch dam with a central abutment practicable. A round-head buttress dam is the most economical. The cost of a solid gravity dam would be 77 per cent. greater, and of a double-arch dam 94 per cent. dearer. It is stated that the cost of the better concrete and more complex shuttering in the buttress dam would be more than offset by the smaller volume of concrete required compared with the gravity dam.

The dam in the second example is 240 ft. high and 930 ft. long. A round-head buttress dam would again be the cheapest, the cost of a solid gravity dam being 88 per cent. greater. The cost of a single-arch dam would be more than twice that of a buttress dam because the valley is not sufficiently narrow for an arch dam of such height to be economical.

The costs of lined and unlined rock tunnels are also considered. It is shown that, in countries where labour is dear and cement comparatively cheap, a tunnel of 11 ft. equivalent diameter lined with concrete would be the most economical. The cost of an unlined tunnel of 16 ft. 6 in. equivalent diameter, with the same hydraulic properties as the lined tunnel, would be 6 per cent. greater, and that of an unlined tunnel, 15 ft. 9 in. equivalent diameter with a concrete invert and the same hydraulic properties, 13 per cent. greater.

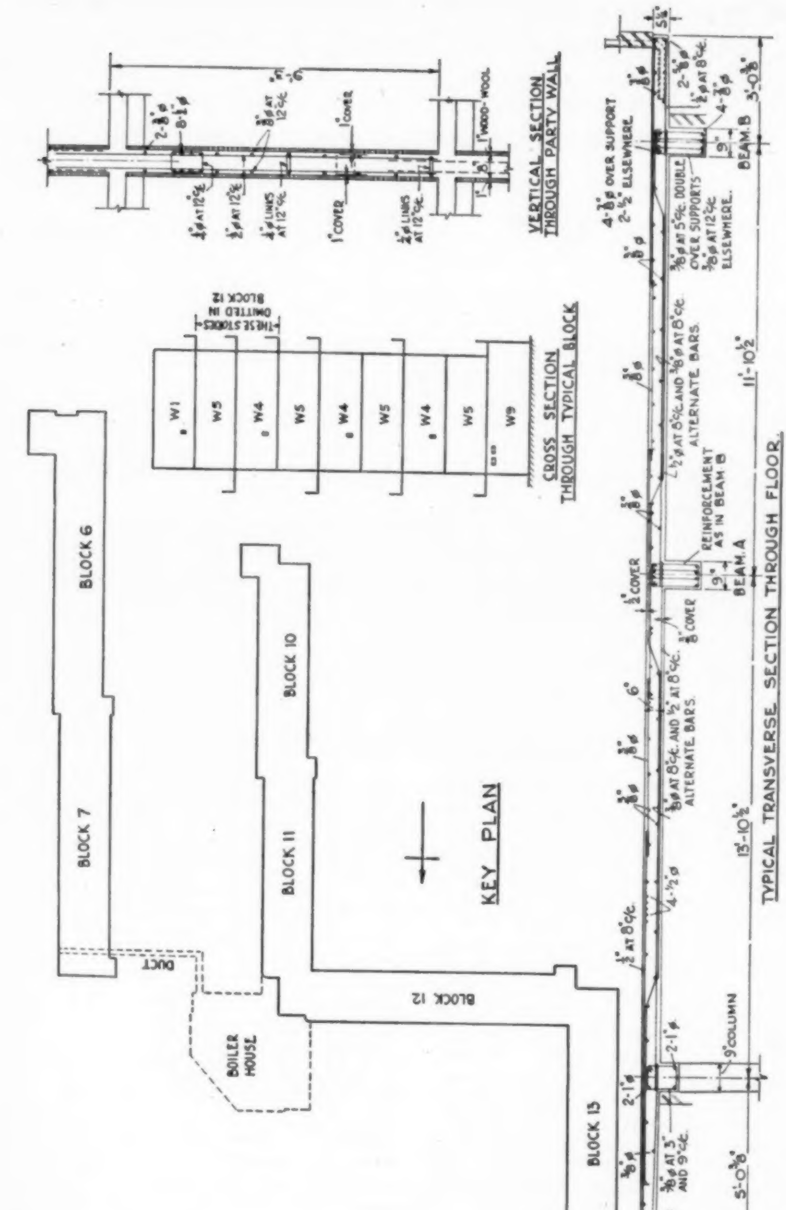


Fig. 2.—Residential Flats in South London.

(See facing page.)

Residential Flats in South London.

SOME features of the reinforced concrete construction of six blocks of residential flats (*Fig.* 1) being built for the Wandsworth Borough Council on the Notre Dame estate, Clapham, are described in the following. The lengths of the buildings, which are arranged as shown in *Fig.* 2, vary from about 130 ft. to 170 ft. There are no expansion joints. The story height is 9 ft. 3 in. Five buildings are of nine stories and are about 90 ft. high, and the other is of seven stories. The width of each building is about 26 ft. excluding the balconies.

The floors, balconies, party walls, stairs, and roofs are of reinforced concrete, and the outer walls of brick. The main load-carrying members are the party walls, which are at 15 ft. centres in four of the structures and at 18 ft. 9 in. centres in the other two, and rectangular columns midway between the party walls along one face of each building. The 6-in. solid floor slabs, a typical cross-section of which is shown in *Fig.* 2, span between three lines of longitudinal reinforced concrete beams which are supported on the party walls and columns. The structures are designed for the superimposed loads specified in the by-laws of the London County Council. The foundation is generally clay on which the bearing pressure does not exceed 3 tons per square foot.

The mixture of the concrete is generally 1 cwt. of ordinary Portland cement to $5\frac{5}{8}$ cu. ft. of uncombined fine and coarse aggregate (1 : $1\frac{1}{2}$: 3 by volume). In the party walls and columns below the first floor the proportions are 1 : 1 : 2. The working compressive stress in the 1 : $1\frac{1}{2}$: 3 concrete is 850 lb. per square inch, and the maximum tensile stress in the mild steel reinforcement is 18,000 lb. per square inch. The average crushing strength of cubes of 1 : $1\frac{1}{2}$: 3 concrete is 4000 lb. per square inch at 28 days.

Fig. 1.

The concrete is mixed in a central mixing plant which comprises steel aggregate bins, which are filled by a crane-grab, a swivel-type weigh-batcher for measuring the aggregates, and a $\frac{1}{2}$-cu. yd. electrically-driven non-tilting drum mixer. The concrete is discharged into cylindrical $\frac{1}{2}$-cu. yd. steel skips which travel on bogies on a short length of rail-track in front of the mixer. The skips are lifted off the bogies by one of several travelling derrick cranes and discharged into mobile hoppers (*Fig.* 3). The hoppers are fixed to the top of a steel frame mounted on four casters so that it can be easily moved. The frame and hopper are lifted from one floor to another by crane.

Party Walls.

The party walls, a detail of which is also shown in *Fig.* 2, are 8 in. thick and are cast between two 1-in. sheets of wood-wool. The reinforcement on each face

is generally ½-in. bars at 12 in. centres vertically and ⅜-in. bars at 12 in. centres horizontally. Additional horizontal bars are provided above openings in the walls. The walls were concreted in two lifts within panels of timber shuttering secured by patent cramps. The concrete is discharged directly into the shutters from three hoppers fixed in a steel frame (*Fig.* 4) which is placed by crane on the floor between two walls. Removable extensions on the chutes from the hoppers enable the concrete to be placed in the two lifts without altering the position of the hoppers. When one wall is concreted the chutes are reversed and the other wall is concreted without moving the frame. Horizontal and vertical gaps are left at intervals between the sheets of wood-wool (*Fig.* 5) so that the face of the concrete can be inspected after the shuttering has been removed.

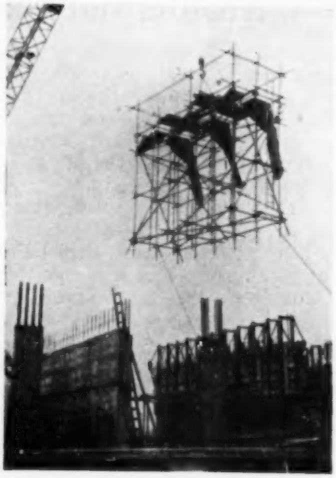


Fig. 4.

Precast Stairs.

The main stairs are cast in situ, but others were precast on the site. Each flat is on two stories, access to the upper rooms being by means of a staircase of thirteen steps the flight being cast as one piece in a timber mould with the soffit uppermost. Eyes embedded in the casting enable it to be lifted by crane and placed in position, where it is temporarily supported by a timber frame (*Fig.* 5) until the beam at the head of the flight has been cast.

The staircases for general use comprise two flights of eight steps each and a landing on each floor. The landing slabs are cast in situ, but the stairs are precast. *Fig.* 6 shows the method of retaining the bottom of the stairs by a 3-in. by 1½-in. steel channel grouted into holes in the walls of the stair well.

Boiler-house Floor.

The boiler house (*Fig.* 2) is entirely below ground. The reinforced concrete

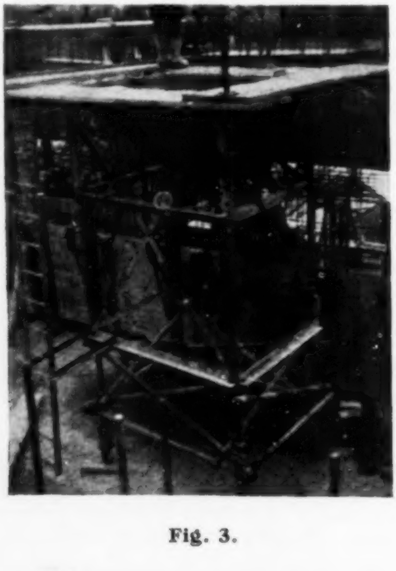


Fig. 3.

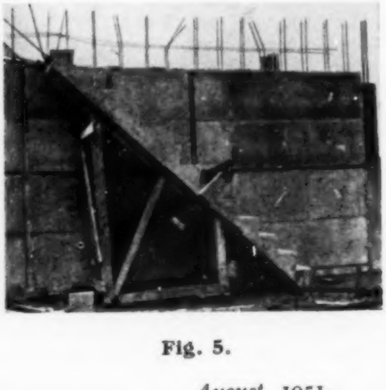


Fig. 5.

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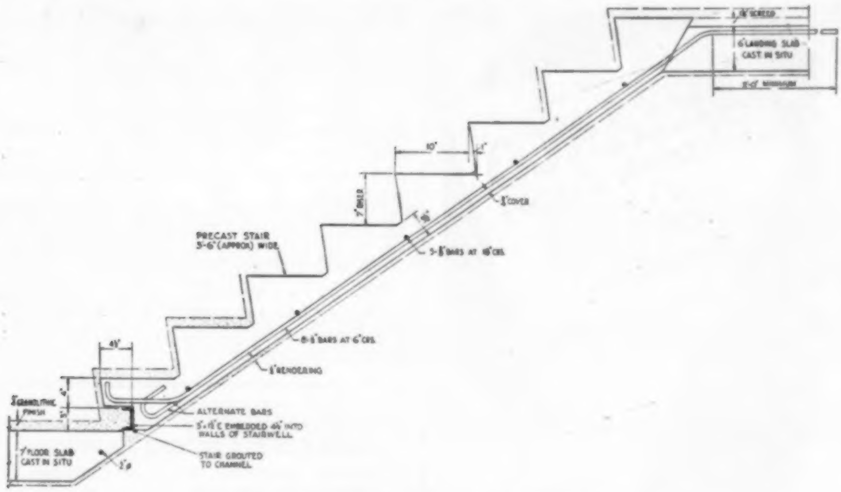


Fig. 6.—Details of Precast Stairs.

retaining wall is 2 ft. to 3 ft. thick and about 15 ft. high; it is not waterproofed. On the inner side there is a 4½-in. brick lining separated from the concrete by a 4½-in. cavity. The construction of the floor (*Fig.* 7) allows for the drainage of any moisture that may seep through the concrete wall and trickle down the cavity. The floor comprises a 9-in. slab of 1 : 8 concrete on the ground or on the base of the retaining wall, 18-in. triangular precast slabs covered with bituminous felt, a 6-in. reinforced concrete slab of 1 : 2 : 4 concrete, and 1½-in. finish. The precast slabs (*Fig.* 7) are generally 1½ in. thick but have pads at each corner so that they

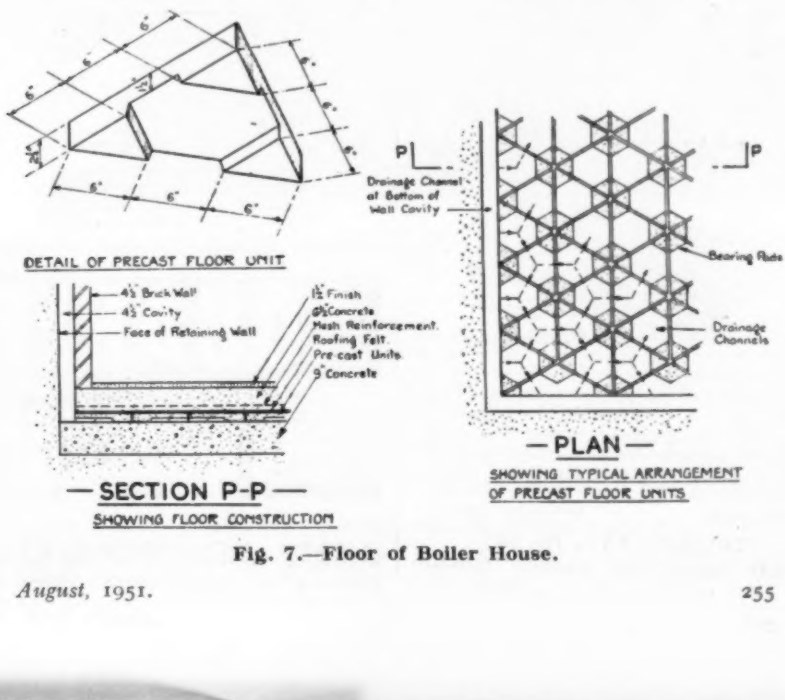


Fig. 7.—Floor of Boiler House.

stand clear of the 9-in. slab and form a series of horizontal channels having direct connection to the bottom of the cavity in the wall and draining to a sump.

The architects are Messrs. James & Bywaters, and the consulting civil engineers Messrs. R. T. James & Partners. The contractors are Messrs. Wm. Moss & Sons, Ltd.

Change of Address.

Mr. R. F. Galbraith, B.Sc., consulting engineer, has removed to 9 Southampton Place, London, W.C.1. (Telephone: Chancery 2067).

A Prestressed Concrete Lighthouse.

THE Berck lighthouse near Boulogne has been reconstructed in prestressed concrete (*Fig.* 1). The structure is 131 ft. 3 in. high to the focal plane, and the shaft is a true cylinder of 18 ft. $4\frac{1}{2}$ in. external diameter.

The cast-in-situ concrete plinth extends 18 ft. 6 in. above the ground and has an external diameter of 20 ft., the wall being 1 ft. 6 in. thick. Thirty-six prestressing cables of eighteen wires each (*Figs.* 1 and 2) extend from the top of the plinth into the foundation, where they are anchored by a U-bend. The tops of these cables are anchored in cones as shown in *Fig.* 2.

The shaft is 106 ft. 3 in. high above the plinth and is constructed of precast concrete segments, which are generally not reinforced and are 7 in. thick, $13\frac{3}{4}$ in. high, and 19 in. long (measured around the circumference). There are thirty-six segments in one course. Special segments are provided at windows and to contain the prestressing cables in the shaft. There are four sets of nine vertical

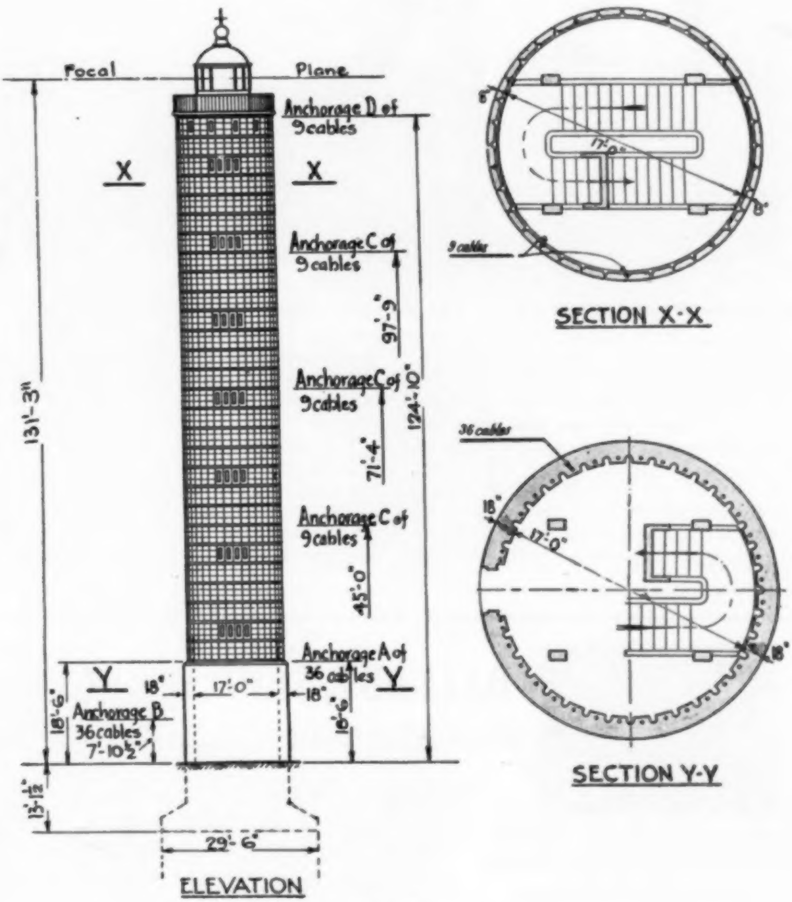


Fig. 1.

cables in the shaft, each cable comprising twelve wires (*Fig.* 2) and extending 10 ft. 8 in. into the plinth. Each set of cables ends at a different height above the plinth as shown in *Fig.* 1. A precast prestressed concrete ring, 4 in. thick, 18 ft. 4½ in. external diameter, and 8 in. wide, is provided between every third which the wires pass, were arranged in a circle. At each block a screw-jack was placed radially to bear against a piece of timber between the block and the jack. The jacks stretched the wires and concrete was placed between the blocks. When the concrete had hardened, the jacks were released and the concrete ring

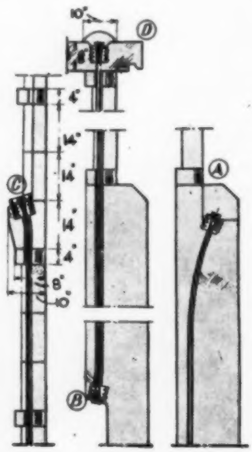


Fig. 2.—Details of Anchorage.

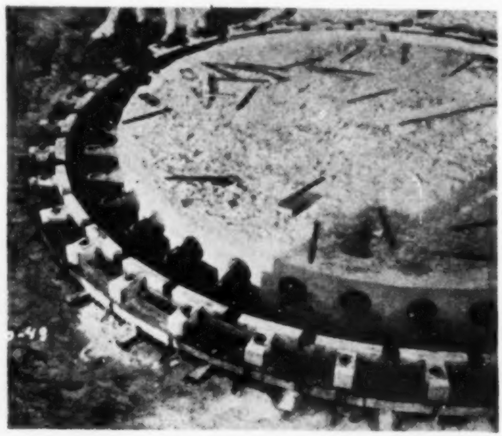


Fig. 3.—Precasting the Prestressed Rings.

course of segments. The segments and rings are set in cement mortar. The wires in the cables are 0·2 in. diameter, and the cables are stretched in situ and anchored in accordance with the Freyssinet system. The rings are prestressed with 0·1-in. wires bonded to the concrete, and were cast as shown in *Fig.* 3. A central circular buttress was constructed and precast rectangular concrete blocks, through was compressed circumferentially. Holes through the blocks permit the passage of the vertical cables after erection of the rings.

The internal reinforced concrete staircase was constructed at the same time as the wall, and the four columns supporting the stairs were used temporarily to support a working platform. These notes and illustrations are from "Travaux."

(Continued on facing page.)

MISCELLANEOUS ADVERTISEMENTS

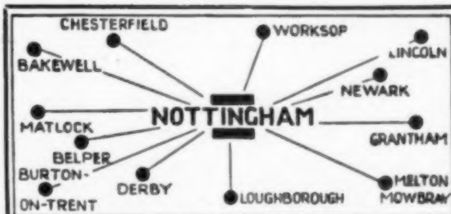
(*Continued from page* lii.)

SITUATION WANTED.

FOR SALE.

Construction work in the Belgian Congo.

THE Societe des Forces Hydro-electriques du Bas-Congo (30 rue Marie de Bourgogne, Brussels, Belgium) wishes to receive tenders for construction work in connection with hydro-electric schemes at Zongo and Tshopo in the Belgian Congo. Information on the work will be sent on request to firms interested, and tender forms will be available later.



DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

AGGREGATES (per cu. yd.).—Washed sand, 17s. Clean shingle: ¾ in., 15s. 2d.; ⅜ in., 17s. 6d. Thames ballast, 15s. 9d.

CEMENT (per ton, delivered at Charing Cross).—
Portland cement, 6 tons and upwards, 93s. 9d. 1 ton to 6 tons, 98s. 9d. Paper bags and non-returnable jute sacks included.
Rapid-hardening Portland, 6s. above ordinary Portland.
Aquacrete and 417, 32s. 6d. above ordinary Portland; paper bags included.
Colorcrete (buff, red, and khaki), in 6-ton loads, 135s. 3d.; paper bags included.
Snowcrete, £12 11s. 3d., inc. paper bags.
"Super-Cement," 32s. 6d. per ton above ordinary Portland cement; paper bags included.
High-alumina cement, 1 ton and upwards, 255s. per ton; paper bags 22s. per ton extra.
Snowcem paint, 56s. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1s. 1d.) and No. 94 (price 5d.) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel round bars (per cwt.); ⅝ in. to 2½ in., 28s. 2d. 9/16 in. to ½ in., 29s. ⅜ in., 29s. 8d. ¼ in., 31s.

Materials and Labour.

(Contracts up to £5000. Inc. 10 per cent. profit.)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—
Foundations, 2s. 2d. per cu. ft. Columns, 2s. 5d. per cu. ft. Beams, 2s. 5d. per cu. ft. Floor slabs 4 in. thick, 6s. 10d. per sq. yd.; Do., 5 in., 8s. 7d.; Do., 6 in., 10s. 3d.; Do., 7 in., 12s. Walls 6 in. thick, 10s. 3d. per sq. yd. Add for hoisting 3s. 6d. per cu. yd. above ground floor level. Add for rapid-hardening Portland cement 2s. per cu. yd.

REINFORCEMENT.—Mild steel round bars, including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ⅜ in., 48s. 6d. 7/16 in. to ⅝ in., 43s. 6d. 11/16 in. to 2½ in., 42s.

SHUTTERING AND SUPPORTS.—
Walls, 155s. per square.
Floors (average 10 ft. high), 160s. per square. In small quantities, 1s. 8d. per sq. ft.
Columns, average 18 in. by 18 in. (per sq. ft.), 1s. 8d.; in narrow widths, 2s.
Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2s.; in narrow widths, 2s. 2d.
Raking, cutting, and waste, 5d. per lin. ft.
Labour on splays, 3d. per lin. ft.
Small fillets to form chamfers, 6d. per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3s. per hour (carpenters 2d. a day tool money); Labourers, 2s. 6d.; Men on mixers and hoists, 2s. 7d.; Bar-benders, 2s. 8d.

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